FOUNDATION ENGINEERING 4D5 - CRIB EXAM 2021

PART 1 - SITE INVESTIGATION and SHALLOW FOUNDATIONS

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Q.1.(a)(i)

Wind load:

$$\begin{aligned} H &= p \cdot A_s \\ &= 2 \cdot 10 \\ &= 20 \text{ kN} \end{aligned}$$

Sliding failure criterion:

$$\begin{aligned} H_{ult} &= N_{cH} \cdot A_f \cdot s_u \\ &= 1 \cdot \frac{\pi 2^2}{4} \cdot 100 \\ &= 314.2 \text{ kN} \end{aligned}$$

Factor of safety:

$$\begin{aligned} \text{FoS} &= \frac{H_u lt}{H} \\ &= \frac{314.2}{20} \\ &= 15.71 >> 1 \therefore \text{ SAFE (i.e. } H << H_{ult}) \end{aligned}$$

Q.1.(a)(ii)

Taiebat and Carter (2000) failure envelope for scenario without 'lift off':

$$f = \left(\frac{V}{V_{ult}}\right)^2 + \left[\frac{M}{M_{ult}}\left(1 - 0.3\frac{H}{H_{ult}}\right)\right]^2 + \left(\frac{H}{H_{ult}}\right)^3 - 1$$

Maximum vertical load (other methods using shape factors and strip bearing capacity factors perfectly acceptable):

$$\begin{aligned} V_{ult} &= N_{cV} \cdot A \cdot s_u \\ &= 6 \cdot \frac{\pi 2^2}{4} \cdot 100 \\ &= 1885.2 \text{ kN} \end{aligned}$$

$$\therefore \frac{V}{V_{ult}} = \frac{200}{1885.2} = 0.1$$

Maximum horizontal load:

$$H_{ult} = N_{cH} \cdot A \cdot s_u = 314.2 \text{ kN (Solved in Part 1.(a))}$$

$$\therefore \frac{H}{H_{ult}} = \frac{20}{314.2} = 0.063$$

Maximum moment load assuming effective width is less than the diameter, and approximated here as the side length of a square of equivalent area :

$$
\begin{aligned}
M_{ult} &= N_{cM} \cdot A \cdot B \cdot s_u \\
&= 0.67 \cdot \frac{\pi 2^2}{4} \cdot \frac{2}{2} \sqrt{\pi} \cdot 100 \\
&= 373.0 \text{ kNm}
\end{aligned}
$$

$$
\therefore \frac{M}{M_{ult}} = \frac{Hh}{M_{ult}} = \frac{20 \cdot 10}{373.0} = 0.54
$$

Substitute in values for V/V_{ult}, H/H_{ult} and M/M_{ult} into the failure criterion and determine whether the loading condition lies within ($f < 0$), on ($f = 0$) or outside ($f > 0$) the failure envelope:

$$
\begin{aligned}
f &= 0.1^2 + [0.54\,(1 - 0.3 \cdot 0.063)]^2 + 0.063^3 - 1 \\
&= 0.01 + 0.28 + 0.00039 - 1 \\
&= -0.71 \longrightarrow f < 0 \therefore \text{ SAFE}
\end{aligned}
$$

In conventional factor of safety terms:

$$
\begin{aligned}
\text{FoS} &= \frac{1}{1+f} \\
&= \frac{1}{1 - 0.71} \\
&= 3.45 > 1 \therefore \text{ SAFE (but much less so than pure sliding where combined loading effects were ignored...)}
\end{aligned}
$$

Q.1.(b)

It has been assumed in the pure sliding calculation in part 1.(a) that the foundation is fully rough, such that the full undrained shear strength of the underlying clay is mobilised at failure. This may not be the case, depending on the surface roughness of the material the foundation is manufactured from, and the mineralogy and particle shape of the underlying soil. The potential embedment of the foundation has been ignored, resulting in a conservative estimate of the factor of safety for this mode of failure.

In the failure envelope calculation in part 1.(b) that the foundation is similarly fully bonded to the soil, and that the soil can maintain tension across the interface in the event that the foundation tries to rotate due to the wind loading applied to the road sign. This is the so-called 'without lift off' condition. The failure envelop also assumes that the foundation is not embedded. Accounting for the embedment would result in a slightly higher factor of safety against failure.

Q.1.(c)

The most likely mode of failure is rotation, since M/M_{ult} is significantly larger than V/V_{ult} or H/H_{ult}, indicating that much of the resistance to rotation is being mobilised already, whereas only a small proportion of the resistance to bearing or sliding failure is being mobilised. A small increase in H would not result in sliding failure, but it would be highly likely to result in rotational failure as $M/M_{ult}(= Hh/M_{ult})$ tends to a maxima.

Q.1.(d)

Long term loading would result in dissipation of negative excess pore pressure induced by the tension acting across the foundation-soil interface, resulting ultimately in loss of tension and 'lift off'. An alternative calculation assuming that tension cannot be maintained across the interface - the so-called 'lift-off' condition - could be used to check for this condition, and is likely to yield a lower factor of safety, because the effective area of the foundation will be reduced by loss of contact as 'lift off' occurs.

Comments: This question was answered by around half of the candidates and answered generally well. The most difficult appeared to be in calculation of the ultimate moment capacity in part (a)(ii), where some students calculated moments with units of force because they missed a breadth term from their modification of the moment equation to account for the circular geometry of the foundation. Other candidates that did not miss this detail often used the diameter in place of breadth, but this will lead to an overestimate of moment capacity. All other aspects were answered consistently competently.

Q.2.(a)

From plasticity theory:

$$\tau_{mob} = \frac{q}{N_c}$$
$$\gamma_{mob} = M_c \cdot \frac{w}{D} = 1.35 \cdot \frac{w}{D}$$

Using a power law stress-strain curve after Vardanega and Bolton (2011):

$$\begin{aligned}
\frac{\tau_{mob}}{s_u} &= 0.5 \cdot \left(\frac{\gamma_{mob}}{\gamma_{M=2}}\right)^b \\
\therefore \frac{2q}{N_c \cdot s_u} &= \left(\frac{1.35}{\gamma_{M=2}} \cdot \frac{w}{D}\right)^b \\
\therefore \left(\frac{2q}{N_c \cdot s_u}\right)^{\frac{1}{b}} &= \frac{1.35}{\gamma_{M=2}} \cdot \frac{w}{D} \\
\therefore w &= \frac{\gamma_{M=2}}{1.35} \cdot \left(\frac{2q}{N_c s_u}\right)^{\frac{1}{b}} \cdot D
\end{aligned}$$

Q.2.(b)
Using the expression derived above:

$$\begin{aligned}
w &= \frac{\gamma_{M=2}}{1.35} \cdot \left(\frac{2q}{N_c s_u}\right)^{\frac{1}{b}} \cdot D \\
&= \frac{0.02}{1.35} \cdot \left(\frac{2 \cdot 63.7}{6 \cdot 100}\right)^{\frac{1}{0.6}} \cdot 2 \\
&= 0.0148 \cdot (0.212)^{1.666} \cdot 2 \\
&= 0.0148 \cdot (0.212)^{1.666} \cdot 2 \\
&= 0.0024\text{m} \\
&\approx 2.24\text{mm}
\end{aligned}$$

Q.2.(c)

The solution is approximate because:

- The compatibility factor, M_c, derived as 1.35 by Osman and Bolton (2005), is based on an approximated mechanism resembling that proposed by Prandtl for plane strain conditions;
- The power stress-strain response has been approximated to a database of laboratory experiments by Vardanega and Bolton (2011), so unless site-specific testing is performed, any predictions will be a best-fit to the existing database of data; and
- The methodology is sensitive to the depth at which the stress-strain response of the soil is derived experimentally, which is usually taken as $0.3D$, which is close to the vertical centroid of the Prandtl mechanism.

Q.2.(d)

A value of 6 is given in the Data Book for the Mobilised Strength Design method. Exact values of 5.69 and 6.05 were derived for smooth and rough interfaces by Eason and Shield (1960), implying that the value of 6 given in the Data Book is close to fully rough.

Q.2.(e)

Immediate settlement under rigid circular foundation:

$$
\begin{aligned}
w_{rgd} &= \frac{\pi}{4}\frac{(1-\nu)}{G} q_{avg} a \text{ where} \\
\nu &= 0.5 \text{ (Immediate settlement)} \\
G &= 10,000 \; (G \approx 100 \mathrm{s}_u) \\
A &= \frac{\pi D^2}{4} = \frac{\pi 2^2}{4} = 3.142 \text{ m}^2 \\
q_{avg} &= 63.7 \text{ kPa (Bearing pressure, } q_{avg} = V/A = 200/3.142) \\
\therefore w_{rgd} &= \frac{3.142}{4} \cdot \frac{0.5}{10,000} \cdot 63.7 \cdot 1 = 0.0025 \text{ m} \approx 2.5 \text{ mm}
\end{aligned}
$$

Long term settlement under foundation:

$$
\begin{aligned}
\frac{w_d}{w_u} &\approx \left(\frac{1-\nu_d}{1-\nu_u}\right) = \frac{0.7}{0.5} = 1.4 \\
\therefore w_d &\approx 1.4 \cdot w_u = 1.4 \cdot 2.5 \approx 3.5 \text{ mm}
\end{aligned}
$$

Q.2.(f)

Settlement of foundation overlying bedrock by a depth of H metres with embedment of D metres, assuming foundation width B is equal to the diameter of the foundation, which is 2 m:

$$
\begin{aligned}
w_u &= \mu_0 \cdot \mu_1 \cdot \frac{qB}{E} \text{ where} \\
\mu_0 &= 1 \text{ (D/B = 0; via chart in Data Book)} \\
\mu_1 &= 0.6 \text{ (H/B = 5 and circular foundation; via chart in Data Book)} \\
E &= 2G(1+\nu) = 30,000 \\
\therefore w_u &= 1 \cdot 0.6 \cdot \frac{63.7 \cdot 2}{30,000} = 0.00255 \text{ m} = 2.55 \text{ mm}
\end{aligned}
$$

The long term value will also be 1.4 times the undrained value as per the previous part of the question, which gives 3.57 mm. The elastic settlements accounting for the presence of bedrock are similar to the standard elastic solution because the value of μ_1 is very close to the asymptotic case for the circular foundation curve used on the chart. In other words, in this case the impact of the presence of bedrock is minimal.

Comments: This question was very well answered by most candidates who tackled it. The main mistakes present in some solutions revolved around the appropriate choice of Poisson's ratio in the elastic settlement calculations. Many candidates mixed drained Poisson's ratio with undrained stiffness in their calculations, or calculated long term settlements and referred to them erroneously as immediate.

QUESTION 3

3(a) *Describe the stages in the installation and life of a displacement pile in soft clay and their effect on the total stress, pore water pressure and effective stress of the in-situ soil around the pile.*

The shaft resistance of a driven pile in clay depends on the in situ conditions and the complex changes that take place during installation, excess pore pressure dissipation and subsequent loading (see figure below).

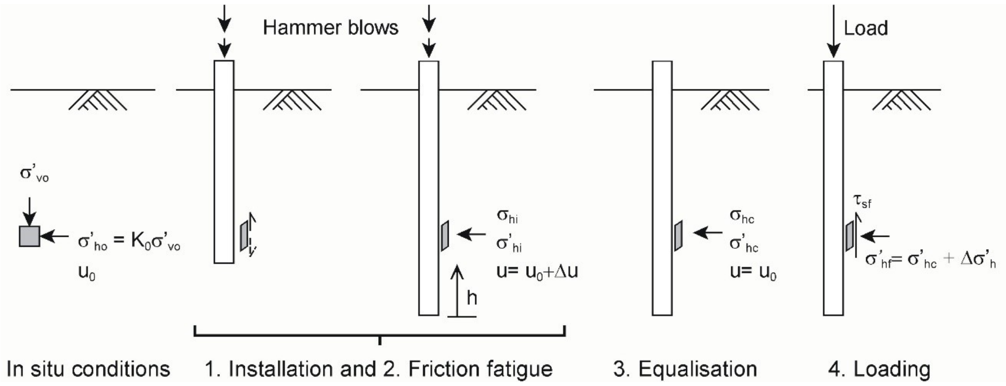

Figure 1: Stages in the installation and life of a driven pile in clay

1) **Installation**: Pile is driven into clay ⇒ high shear strains in soil surrounding pile as the pile tip passes. Process considered undrained (no volume change). Total stress increases ($\sigma_{hi} > \sigma h0$) as soil is pushed away to accommodate pile. Changes in pore pressure arising from (i) increase in total stress and (ii) undrained shearing. Soft clay is contractile ⇒ Trying to expel water ⇒ Positive contribution to excess pore pressure.

2) **Friction Fatigue**: Cyclic shearing of the soil close to pile shaft during installation encourages contraction of the soil. Reduction in total and effective stress on pile surface.

3) **Equalisation**: After installation, there is a field of excess pore pressure around the pile that needs to dissipate. In soft clay : Pressures entirely positive. Results in increase in effective stress and increases available shaft resistance: process called.

4) **Loading**: Small drop (20%) in horizontal effective stress (+ associated small rise in pore pressure) occurs during loading.

A diagram to explain the changes in total stress, effective stress and pore water pressure, in the same format as question 3(a) of the example paper is also welcome as an answer to this question and qualifies to full mark if correct.

Suggested Marking: installation = [5%]; friction fatigue = [5%]; equalisation properly = [5%]; loading = [5%] - TOTAL = [20%]

3(b) *Calculate the driving force required to install the pile by assuming that the pile becomes plugged in the early phases of the installation process. Assume undrained installation conditions with a pile skin friction factor estimated using the ISO/API method of Figure 2.*

This question is similar to question 2 of example paper

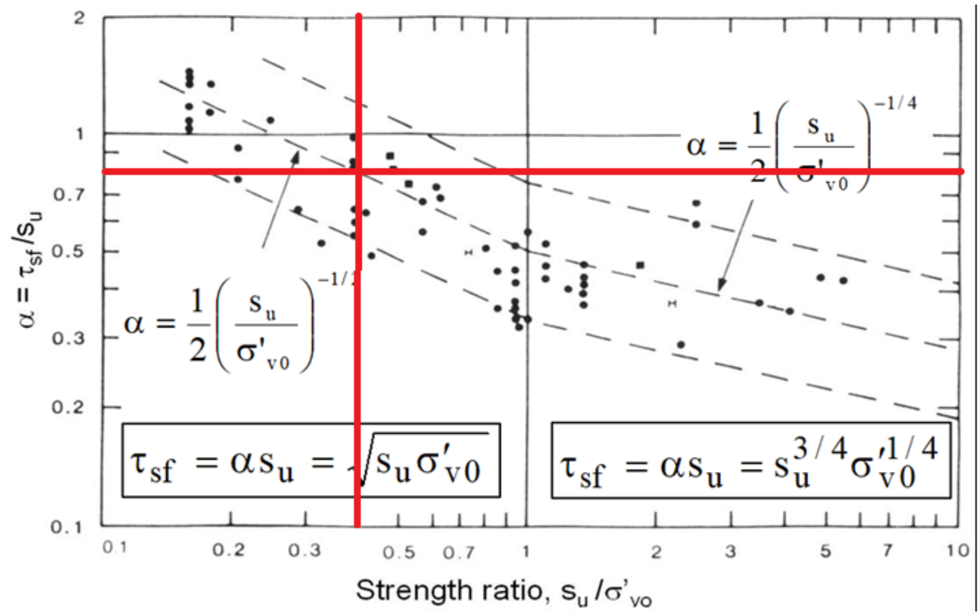

Figure 2: ISO/API skin friction factor estimation for clay soils

Base resistance:

$$Q_b = q_b \frac{\pi D^2}{4} = N_c s_{u,base} \frac{\pi D^2}{4} = 9 \times 2 \times 10 \frac{\pi \times 1^2}{4} = 141.4kN \tag{1}$$

Shaft Resistance: α read from Fig. 2 with:

$$\frac{s_u}{\sigma'_{v0}} = \frac{2 \times z}{\gamma' \times z} = \frac{2}{5} = 0.4 \tag{2}$$

Hence, $\alpha = 0.8$. It follows:

$$Q_s = \pi D L \tau_s = \pi D L \alpha s_{u,mean} = \pi \times 1 \times 10 \times 0.8 \times 0.5 \times 2 \times \frac{10}{2} = 251.3kN \tag{3}$$

And therefore:

$$Q = Q_b + Q_s = 141.4 + 251 = 392.7kN \tag{4}$$

Since the pile is plugged, the following answer is more correct, and was also accepted:

$$Q = Q_b + Q_s + W_p = 141.4 + 251.3 + 34.7 = 427kN \tag{5}$$

where $W_p = \pi \frac{D_i^2}{4} L\gamma' = \pi \frac{(D-2t)^2}{4} L\gamma' = 34.7kN$ is the plug weight.

Suggested Marking: calculate Q_b = [10%]; Calculate s_u/σ'_v = [10%]; use chart = [5%]; calculate Q_s = [10%]; calculate Q = [5%] - TOTAL = [40%]

3(c) *Explain why and how a CPT test could be used to assess the pile axial capacity and what the limitations of this technique are.*

CPT resistance, q_c, is increasingly used to predict pile capacity, since a CPT resembles a miniature pile. q_c can be linked to pile axial capacity such as follows.

How to use the CPT:
(1) *Shaft Resistance*: Data from cone resistance factored down by about 100 (50 to 250 is also accepted), corresponds broadly to the local shear stress recorded along the pile. An estimate of the shaft resistance can hence be obtained from q_c. (2) *Base Resistance*: Design charts can be used to estimate the proportion of cone resistance, q_c, that can be mobilised at a given normalised settlement of the pile base, w_b/D. These curves account for the compression of the soil plug and are a deliberate attempt to estimate the minimum base resistance on a soil plug, to be conservative, and as such ignore residual base load.
(3) *UWA design method*: The UWA design Method also proposes a calculation method to predict ultimate shaft resistance based on q_c.

Limitations:
1- It is necessary to apply a reduction factor to q_c to estimate q_{bf} on the plug of an open-ended pile if settlement cannot be tolerated.
2- Piles are often installed at only a shallow embedment into a strong bearing stratum. A significant embedment- around 8 diameters- is required before the softer overlying layer is no longer "felt" by the tip of a CPT or the base of a pile. Since a pile has a greater diameter than a CPT instrument, a deeper embedment into a hard layer is required to mobilise the "full" strength of that layer. Prior to sufficient penetration, qbf will be less than the local qc since the previous layer will still be "felt" by the pile tip. A simple equation to account for partial embedment by "correcting" q_c to an averaged value, $q_{c,corrected}$, at shallow penetration can be used.

Suggested Marking: CPT=miniature pile + separate shaft and base [5%]; Use = shaft [5%]; Base = [5%]; Mention of UWA method = [5% extra as it is outside curriculum but available for them to read]; Limitation 1 = [5%]; Limitation 2 = [5%]; - TOTAL = [25%]

3(d) *The coefficient of horizontal consolidation for this site is $c_h = 16m^2/year$. How soon after the piles are installed can construction of the building start?*

This question is similar to question 2 of example paper

$$T_{eq} = \frac{c_h \times time}{D_{eq}^2} \Rightarrow time = \frac{T_{eq} D_{eq}^2}{c_h} = \frac{T_{eq} \times 4Dt}{c_h} = 0.075year = 27days \tag{6}$$

Note: D_{eq} is obtained from lecture notes and is equal to $D_eq = 2\sqrt{Dt} = 2 \times \sqrt{1 \times 0.03} \Rightarrow D_{eq}^2 = 4Dt = 0.12m$

Suggested Marking: Expression for D_{eq} = [5%]; Calculation = [10%]; - TOTAL = [15%]

Comments: This question was answered by around half of the candidates and was very well answered by most candidates who tackled it. Question 3(a) was well succeeded but most students did not describe the change in stresses during the loading phase. Question (b) was well succeeded. Fig. 1 of the exam paper had a mistake on the expression for τ_{sf} which is not consistent with the formulae for α just above (and in the data book). This Figure was extracted from the lectures (and will be amended in future years). 2 candidates used the wrong formulae but were given full marks as an account for the mistake in the paper; most candidates answered this question very well. Question (c) was the most difficult question, with most students stating general knowledge about CPT but not relating it to the specifics of pile capacity assessment. Question (d) was succeeded by about half of the students, with most errors coming from the calculation of D_{eq}.

QUESTION 4

4(a) *Calculate the ultimate lateral force per unit length p_u assuming a bearing factor of $N_c = 9$ at all depth.*

Formulae given in the data book p. 17.

$$p_u = N_c s_u D = 9 \times 120 \times 0.8 = 864kN/m \tag{7}$$

Suggested Marking: TOTAL = [5%]

4(b) *Assume that the pile fails as a rigid body. Calculate the depth of the point of rotation, with the origin taken at the ground surface, and calculate the ultimate lateral load H_{ult}.*

Similar to question 14 in example paper

Horizontal equilibrium:

$$H_{ult} = p_u z_{rot} - p_u(L - z_{rot}) \tag{8}$$

Moment equilibrium about head:

$$(p_u z_{rot})\frac{z_{rot}}{2} = p_u(L - z_{rot})\left(\frac{L + z_{rot}}{2}\right) \tag{9}$$

$$\Rightarrow \frac{z_{rot}^2}{2} = \frac{L^2}{2} - \frac{z_{rot}^2}{2} \Rightarrow \frac{z_{rot}}{L} = \frac{1}{\sqrt{2}} \tag{10}$$

Hence,:

$$z_{rot} = 7.1m \tag{11}$$

Replacing in $H_{ult} = p_u\,(2z_{rot} - L)$:

$$H_{ult} = 3.6MN \tag{12}$$

Suggested Marking: Horizontal equilibrium = [10%]; moment equilibrium = [10%]; solve = [5%]; calculate H_{ult} = [10%] . TOTAL = [35%]

4(c) *What is the required pile wall thickness to prevent structural failure? Note: plastic capacity $M_p = \sigma_y D^2 t$.*

A hinge will form at maximum bending moment, i.e. when the shear force equal 0 ($\frac{dM}{dx} = SF$)

From equilibrium of forces for the long pile mechanism:

$$H_{ult} = p_u \times z_{SF=0} \Rightarrow z_{SF=0} = \frac{H_{ult}}{p_u} = 4.1m \tag{13}$$

Maximum bending moment in pile using equilibrium of moments:

$$M_{max} = H_{ult} z_{SF=0} - p_u \frac{z_{SF=0}^2}{2} = 7.4MNm \tag{14}$$

This gives a required wall thickness of:

$$t = \frac{M_{max}}{\sigma_y D^2} = 33mm \tag{15}$$

Suggested Marking: Location of hinge SF=0 = [7.5%]; calculate z_{crit} = [7.5%]; moment equilibrium = [10%]; calculate M_{max} and t = [10%] . TOTAL = [35%]

4(d) *Reducing the pile thickness to 22 mm enables significant cost saving. What is the maximum lateral load that can be applied on this pile before it fails structurally?*

Calculate new value of M_p:

$$M_p = \sigma_y D^2 t = 4.9MNm \tag{16}$$

Use design diagram for long pile mechanism in data book p. 17 (Figure 3) with:

$$\frac{M_p}{s_u D^3} = 80 \tag{17}$$

This gives:

$$\frac{H_{ult}}{s_u D^2} = 30 \Rightarrow H_{ult} = 2.3MN \tag{18}$$

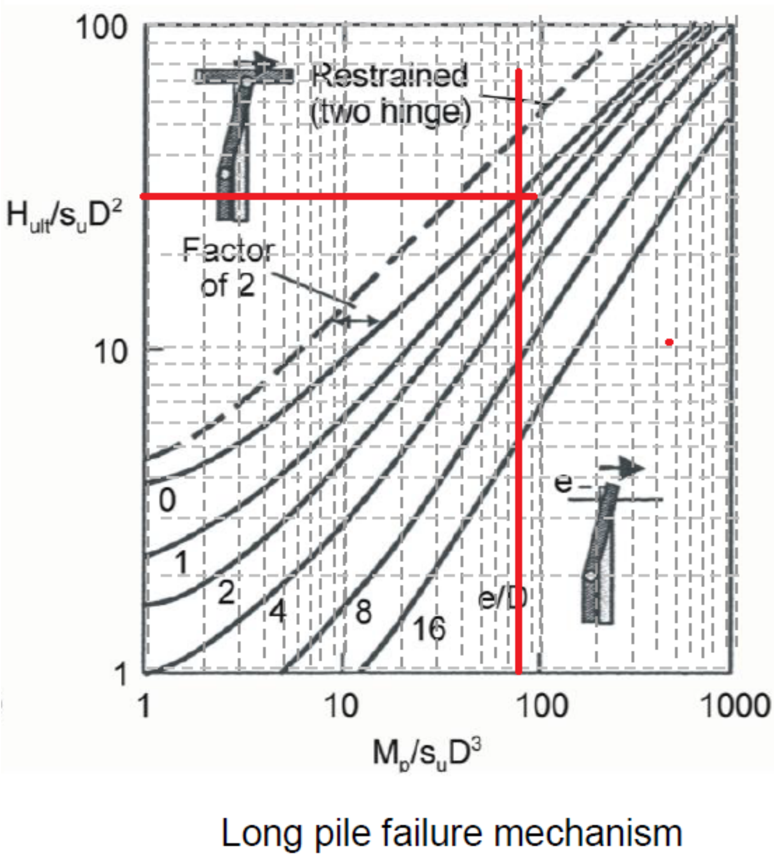

Figure 3: Design diagram for long pile failure in uniform clay

Suggested Marking: Calculate M_p/s_uD^3 = [7.5%]; Use design diagram = [10%]; calculate H_{ult} = [7.5%] . TOTAL = [25%]

Comments: This question was attempted by all students and well answered overall. The difficulty was to choose the appropriate method for calculations of the moment in question (b). Candidates who attempted the diagram method instead of resolving equilibrium obtained an erroneous answer because of the lack of precision of the diagram for this L/D ratio. A few candidates used the wrong diagram in question (d).